An aseismic design method for preventing damage of a soil-pile foundation-superstructure system

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Abstract

An effective aseismic design method for the conditions of ground improvement is proposed for preventing liquefaction of ground and reducing damage of pile foundations. The proposed method uses two existing computer programs; NUW2 for 2D effective stress analysis and WAP3 for simulation of the sand compaction pile method. The analysis is extended to take the soil-pile interaction into consideration. Numerical computations for responses of the ground and the pile foundations are performed for the 22 ground models in the Kobe area. By using the natural period of the ground as the key parameter, the critical conditions of ground improvement are evaluated and related with the responses of pile and superstructure on the ground. The main conclusion is that the proposed method is effective for reducing pile stress under an allowable limit by ground improvement in sandy layers, but the ground improvement is difficult to prevent pile failure in soft clayey ones.

Keywords: effective stress analysis, liquefaction, pile, ground improvement, interaction, aseismic design.

1 Introduction

Many damages of foundation structures and tilt of structures occurred at reclaimed land during 1995 Hyogo-ken-nanbu Earthquake (AIJ [1]). The characteristics of surface ground layers by strong ground motion and liquefaction causes the damages of pile foundation. There are many countermeasures against
liquefaction, which have been developed and conducted in the field, but a few of them have been investigated its efficiency for preventing both liquefaction and damage of the structure resting on or buried in the ground. Thus it is important to evaluate the non-linear response of the surface ground layers surrounding structure including liquefaction and reflect it for the response of the structure constructed on or in the surface ground layers.

In this study we propose the effective aseismic design method which evaluates the ground improvement condition for preventing liquefaction (Akiyoshi et al. [2]) and damage of pile foundation.

2 Seismic response analysis method for soil–pile system

Figure 1 shows the analytical model which consists of surface ground layers and pile foundations system and the single degree of freedom structure (SDOF) on the surface. The analytical method consists of existing computer programs which are the effective stress analysis program “NUW2” (Akiyoshi et al. [3]) and the simulation of the compacting ground improvement “WAP3” (Akiyoshi et al. [4]). The program “NUW2” is based on Biot’s two-phase mixture theory and Iai’s constitutive equation (Iai et al. [5]). The program “WAP3” simulates the static and dynamic compaction process of sand compaction pile method and is based on the accumulation of propagating waves by compaction.

Combining these two programs, seismic responses of the ground and piles are evaluated for the several conditions of ground improvement and natural periods of structures on surface ground. In this study we pay attention to the natural period of the surface ground layers $T_G$ as key parameters for evaluating above responses for preventing liquefaction and damage of pile foundations. The natural period of the surface ground layers $T_G$ is calculated by using the S wave velocity $V_{si}$ and thickness $H_i$ of i-th layer into equation (1).

![Figure 1: Soil-Pile-SDOF structure model.](image-url)
\[ T_G = 4 \sum_{j=1}^{n} \left( H_j / V_{st} \right) \]  

Following above analysis, the analysis of dynamic interaction between soil and pile is also introduced. The soil-pile-superstructure system (BIPS: base isolated pile system) is modeled by the lumped masses and springs. The results of seismic motions such as accelerations, velocities and displacements calculated by NUW2 are inputted to the masses through the soil springs. The governing equations of the pile and the superstructure are written as the following ones.

### Piles

\[
\begin{bmatrix} m_i \dot{y}_i \\ C_{ij} \dot{y}_j - u_i^s \\ K_{ij} \dot{y}_i - u_i^s \\ K_{ij} \ddot{y}_i \end{bmatrix} = - \ddot{u}_g \begin{bmatrix} m_i \end{bmatrix} \{ I \} \]  

### Base Foundation

\[
m_n \ddot{y}_n + C_B (\dot{y}_n - u_n^s) + T \left\langle K_{nj} \right\rangle \{ y_n - u_n^s \} + T \left\langle K_{nj} \right\rangle - F_B (y_{n+1} - y_n) = -m_n \ddot{u}_g \]  

### 1st Floor Base

\[
m_{n+1} \ddot{y}_{n+1} + C_n (\dot{y}_{n+1} - \dot{y}_n) + F_n (y_{n+1} - y_n) - C_{n+1} (\dot{y}_{n+2} - \dot{y}_{n+1}) - k_{n+2} (y_{n+2} - y_{n+1}) = -m_{n+1} \ddot{u}_n \]  

### Superstructure

\[
m_i \ddot{y}_i + C_i (\dot{y}_i - \dot{y}_{i-1}) + k_i (y_i - y_{i-1}) - C_{i+1} (\dot{y}_{i+1} - \dot{y}_i) - k_{i+1} (y_{i+1} - y_i) = -m_i \ddot{u}_i \]  

### Whole Piles

\[
\begin{bmatrix} Q \\ M_{1-n} \end{bmatrix} = \begin{bmatrix} A & B & C \\ D & E & F \end{bmatrix} \begin{bmatrix} y \\ \theta \end{bmatrix} \]  

where, \( y \)=pile displacement, \( \theta \)=pile deflection angle, \( Q \)=shear force, \( M_i \)=bending moment, \( \{ m_i \} \)=mass matrix, \( \{ C_{ij} \} \)=damping matrix, \( K^a \)=stiffness of surrounding soil, \( K^p \)=stiffness of pile, \( F \)=elastic force, \( T \)=total number of piles.

If we consider boundary conditions \( M=0 (M_1=...M_n=0) \) and \( \theta_n =0 \)(Fixed pile head),

\[
\theta = -E^{-1}Dy - E^{-1}F\theta_n = -E^{-1}Dy \quad , \quad Q = K^p y \]  

where \( K^p = A - BE^{-1}D - K_{\ddot{y}} \)

The governing equation of soil-pile system is written as

\[
M\ddot{Y}_{t+\Delta t} + C\dot{Y}_{t+\Delta t} + KY_{t+\Delta t} = -M\ddot{Z}_{t+\Delta t} + C^S\dot{U}_{t+\Delta t} + K^SU_{t+\Delta t} + F(Y_{t+\Delta t}) \]  

where, \( C^S \)=damping matrix of structure, \( Z \)=base displacement of soil, \( U \)=displacement of soil, \( Y \)=displacement of pile.

Figure 2 shows the proposed design flow of ground improvement conditions for preventing liquefaction and damages of pile foundations.

### 3 Results of numerical computations and consideration

#### 3.1 Model of surface ground layers and pile foundation system

Figure 3 shows the damage examples of pile foundations of buildings during 1995 Hyogo-ken-nanbu Earthquake (AIJ [6], Seo et al. [7]). The examples are
represented as symbols according to both natural periods of structures and ground layers, and the black triangle symbols show the examples in which both liquefaction and damages of pile foundations occurred. From these damage examples we choose the 22 examples and prepare the computational models of them. In each model the surface ground layers are divided to the 2-dimensional finite elements mesh at pile space and intervals of 2m for both horizontal and vertical directions, respectively. The strong ground motion record (NS component) at Port Island in depth GL-32m in 1995 is used as the input seismic wave with maximum acceleration 5.4m/s². Pile foundations are assumed to be arranged in square shape distribution, and its geometrical moment of inertia and sectional area per unit length are also assumed to be equivalent to the sectional area of each building and total number of piles. In NUW2, piles are modeled by beam elements with linearly elastic characteristics and no relative displacement between pile and soil, and fixed rotation at pile head are assumed.

![Diagram of proposed aseismic design method]

**Figure 2:** Flow of proposed aseismic design method.

In the case of ground improvement by sand compaction pile (SCP) method, the surface ground layer models of above examples are improved by the same conditions of SCP as the reference Akiyoshi [4], and the responses of them are also analyzed.

Piles are assumed to be concrete piles (AC pile, PC pile and PHC pile) or steel piles which have the allowable strengths as 7840kPa and 156800kPa, respectively.
3.2 Effects of ground improvement

Figures 4(a) and 5(a) show the vertical distributions of SPT-N values as a parameter of the compacting time from 0sec to 150sec per one stage of lift up of casing pipe in 1m for the cases of the damage example No.15 and 16, respectively. Figures 4(b) and 5(b) show the averaged natural period of ground layers versus the compacting time. Figures 4(c) and 5(c) show the liquefaction potential PL versus the natural period of ground.

In Figure 4 (a) and (b), as the compacting time becomes long, the SPT-N values increase and the natural periods of ground layers decrease. The horizontal line of the critical limit of liquefaction in Figure 4(b) and (c) means that no liquefaction occurs for the range of the natural period of the ground $T_G$ less than 0.88sec because the index of liquefaction potential $P_L$ (JRA [8]) is under 5.0 within this range. This suggests that the compacting time 10sec per one stage is enough to prevent liquefaction for this example No.15.

In the case of the damage example No.16 in Figure 5, the SPT-N values are also improved effectively except in the soft layer from the depth GL-13m to 20m. As a result of these N values after improvement, the natural period of the ground at the liquefaction limit is obtained as 0.66sec and the condition of compacting time is needed only 20sec.
3.3 Results of liquefaction analysis

Figure 6 and 7 show the vertical distributions of maximum responses of the damage example No.15 and 16, respectively. Both Figures (a) and (b) show the
response of the pile displacement and the bending stress, respectively. In Figure 6 for the example No.15, the distribution of pile displacement of the improved cases changes largely at the depth of GL-17m and 26m where are near the boundaries between soft and hard soil layer. The bending stress concentrates at the same depths and is larger than the allowable strength. In the case of the example No.16 in Figure 7, the pile displacements near the surface are slightly reduced by the improvement. The bending stresses concentrate at the depth GL-12m and 17m where the displacement changes very much because of near the boundary of different soil layers.

Figure 6: Maximum responses of ground of damage example No.15.

Figure 7: Maximum responses of ground of damage example No.16.

Figure 8 shows the distributions of maximum displacements for the example No.15, and (a) and (b) show the cases of no improvement and improvement by the compacting time 150sec, respectively. In both figures 8(a) and (b), the results of no interaction and soil-pile interaction are shown as the symbols of rectangle and circle, respectively. The displacements of soil-pile interaction analysis (BIPS) are larger than those of no interaction case (NUW2) at near the surface.

Figure 9 shows the distributions of maximum bending stress for the example No.15, and, (a) and (b) and symbols mean the same ones as Figure 8. The bending stress of NUW2 in Figure 9(a) and (b) is larger than the allowable value
for the several part of the depth. The bending stresses of the soil-pile interaction case (BIPS) are lower than the allowable one for almost whole depth, but that is still larger than the allowable one at near the surface.

Figure 8: Maximum displacement (No.15).

Figure 9: Maximum bending stress (No.15).

Figure 10: Maximum displacement (No.16).

Figure 11: Maximum bending stress (No.16).

Figures 10 and 11 show the distributions of maximum displacements and bending stresses for the example No.16, respectively, and in both Figures (a), (b) and symbols mean the same ones as Figure 8 and 9. The displacement and
bending stress responses in the case of interaction analysis (BIPS) are larger than that of no interaction case (NUW2) for almost part of the depth, but the bending stresses are smaller than NUW2 at the depth GL-12m and 17m where the bending stress of NUW2 of improved case is larger than allowable one. This means that the aseismic design is successful for the example No.16.

Table 1 shows the possibility of preventing liquefaction and damage of pile with the distribution of clay layer for the 22 damage example cases. There are 8 cases of examples which prevent both liquefaction and damage of pile. There are 7 cases of examples which prevent liquefaction but fail to prevent damage of pile. Thus it is important to develop the countermeasure for the cases in which fail in preventing both of liquefaction and damage of pile.

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4 Conclusions

In this paper the aseismic design method for preventing liquefaction and damage of pile foundation is proposed and the design conditions of ground improvement are investigated by the natural period of the ground as key parameter. The main conclusions are summarized as follows:

(1) Effect of ground improvement by sand compaction pile method is possible to be evaluated by natural period of surface ground layers. This natural period of ground is related to the compacting time which is one of the conditions of ground improvement method.
(2) The method of determining the natural period within preventing damage of pile is reasonable one and possible to be included to the proposed design method.

(3) The proposed design method is applied to 22 damage examples of piles in 1995 Hyogo-ken-nanbu Earthquake and there are 8 examples which prevent both liquefaction and damage of piles. But there exist also 6 examples which cannot prevent both liquefaction and pile damage.

(4) The proposed method is difficult to apply to the surface ground layers which include thick clay layer.

References


